CHAPTER 7

EARTHQUAKE RESPONSE ANALYSIS

7-1. Introduction. A dynamic method of analysis is required to properly assess the safety of existing concrete arch dams and to evaluate proposed designs for new dams that are located in regions with significant seismicity. Dynamic analysis is also performed to determine the adequacy of structural modifications proposed to improve the seismic performance of old dams. The prediction of the actual dynamic response of arch dams to earthquake loadings is a very complicated problem and depends on several factors including intensity and characteristics of the design earthquakes, interaction of the dam with the foundation rock and reservoir water, computer modeling, and the material properties used in the analysis. Detailed descriptions of the recommended dynamic analysis procedures are provided in the "Theoretical Manual for Analysis of Arch Dams" (Ghanaat 1993b). Guidance concerning the seismic studies needed to specify the design earthquake ground motions, methods of analysis, parameters influencing the dam response, and the presentation and evaluation of the analysis results are discussed in this chapter.

7-2. Geological-Seismological Investigation. Estimation of appropriate seismic excitation parameters is an important aspect of the seismic design, analysis, and evaluation of new and existing dams. Concrete arch dams built in seismic regions may be subjected to ground shaking due to an earthquake at the dam site or, more likely, to ground motions induced by distant earthquakes. In addition, large dams may experience earthquakes triggered at the dam site immediately following the reservoir impoundment or during a rapid drawdown. However, such reservoir-induced earthquakes are usually no greater than those to be expected without the reservoir, and they do not augment the seismicity of the region. The estimation of future earthquake ground motions at a dam site requires geological, seismological, geophysical, and geotechnical investigations. The primary purposes of these studies are to establish the tectonic and geologic setting at and in the vicinity of the dam site, to identify active faults and seismic sources, to collect and analyze the historic and instrumental seismic data, and to study the foundation conditions at the dam site that form the basis for estimating the ground motions. However, the lack of necessary data or difficulty in obtaining them, as well as numerous uncertainties associated with the source mechanism and the seismic wave propagation, often complicate the estimation process of ground motions. Therefore, at the present time seismic parameters for dam projects are approximated by empirical relations and through simplified procedures that decouple or neglect the effects of less understood phenomena. The primary factors that must be considered in determination of the seismic parameters for dam projects are discussed in the following paragraphs.

a. Regional Geologic Setting. A study of regional geology is required to understand the overall geologic setting and seismic history of a dam site. The study area, as a minimum, should cover a 100 km radius around the site. But in some cases it may be extended to as far as 300 km in order to include all significant geologic features such as major faults and to account for area-specific attenuation of earthquake ground motion with distance. A typical geologic study consists of:

(1) Description of the plate tectonic setting of the dam region together with an account of recent movements.

(2) Regional geologic history and physiographic features.

(3) Description of geologic formations, rock types, soil deposits.

(4) Compilation of active faults in the site region and assessment of the capability of faults to generate earthquakes.

(5) Characterization of each capable fault in terms of its maximum expected earthquake, recurrence intervals, total fault length, slip rate, slip history, and displacement per event, etc. Field work such as an exploratory trench or bulldozer cuts may also be required to evaluate the seismic history.

b. Regional Seismicity. The seismic history of a region provides information on the occurrence of past earthquakes that help to identify seismicity patterns and, thus, give an indication of what might be expected in the future. Procedures for estimating the ground motion parameters at a particular site are primarily based on historic and instrumentally recorded earthquakes and other pertinent geologic considerations. It is important, therefore, to carefully examine such information for accuracy, completeness and consistency. When possible, the following investigations may be required:

(1) Identification of seismic sources significant to the site, usually within about a 200-km radius.

(2) Development of a catalog of the historical and instrumentally recorded earthquakes for the dam site region. The data, whenever possible, should include locations, magnitudes or epicentral intensity, date and time of occurrence, focal depth, and focal mechanism.

(3) Illustration of the compiled information by means of appropriate regional and local seismicity maps.

(4) Analysis of seismicity data to construct recurrence curves of the frequency of earthquakes for the dam region, to examine spatial patterns of epicenters for possible connection with the identified geologic structures, and to evaluate the catalog for completeness and accuracy.

(5) A review of the likelihood of reservoir induced seismicity (RIS) at the dam site, although this is not expected to influence the design earthquake parameters as previously mentioned.

c. Local Geologic Setting. Local geology should be studied to evaluate some of the site-specific characteristics of the ground motion at the dam site. Such data include rock types, surface structures, local faults, shears and joints, and the orientation and spacing of joint systems. In some cases, there may be geologic evidence of primary or sympathetic fault movement through the dam foundation. In those situations, a detailed geologic mapping, and geophysical and geotechnical exploration should be carried out to assess the potential, amount, and the type of such movements at the dam site.

7-3. Design Earthquakes. The geological and seismological investigations described in the previous paragraph provide the basis for estimating the earthquake ground motions to be used in the design and analysis of arch dams. The level of such earthquake ground motions depend on the seismic activity in the dam site vicinity, source-to-site distance, length of potential fault ruptures, source mechanism, surface geology of the dam site, and so on. Two approaches are available for estimating the ground motion parameters: deterministic and probabilistic. Both approaches require specification of seismic sources, assessing maximum magnitudes for each of the sources, and selecting ground motion attenuation relationships. The probabilistic analysis requires the additional specification of the frequency of earthquake recurrence for each of the sources in order to evaluate the likelihood of exceeding various level of ground motion at the site. The earthquake ground motions for which arch dams should be designed or analyzed include OBEs and MDEs. The ground motions defined for each of these earthquakes are discussed in the following paragraphs.

a. Operating Basis Earthquake (OBE). The OBE is defined as the ground motion with a 50 percent probability of being exceeded in 100 years. In design and safety evaluation of arch dams, an OBE event should be considered as an unusual loading condition as described in Chapter 4. The dam, its appurtenant structures, and equipment should remain fully operational with minor or no damage when subjected to earthquake ground motions not exceeding the OBE.

b. Maximum Design Earthquake (MDE). The MDE is the maximum level of ground motion for which the arch dam should be analyzed. The MDE is usually equated to the MCE which, by definition, is the largest reasonably possible earthquake that could occur along a recognized fault or within a particular seismic source zone. In cases where the dam failure poses no danger to life or would not have severe economic consequences, an MDE less than the MCE may be used for economic reasons. An MDE event should be considered as an extreme loading condition for which significant damage is acceptable, but without a catastrophic failure causing loss of life or severe economic loss.

c. Reservoir-induced Earthquake (RIE). The reservoir-induced earthquake is the maximum level of ground motion that may be triggered at the dam site during filling, rapid drawdown, or immediately following the reservoir impoundment. Statistical analysis of the presumed RIE cases have indicated a relation between the occurrence of RIE and the maximum water depth, reservoir volume, stress regime, and local geology. The likelihood of an RIE is normally considered for dams higher than about 250 feet and reservoirs with capacity larger than about 10^5 acre-feet, but the possibility of an RIE occurring at new smaller dams located in tectonically sensitive areas should not be ruled out. The possibility of RIE's should therefore be considered when designing new high dams, even if the region shows low historical seismicity. The determination of whether the RIE should be considered as a dynamic unusual or a dynamic extreme loading condition (Table 4-2) should be based on the probability of occurrence but recognizing that the RIE is no greater than the expected earthquake if the reservoir had not been built.

7-4. Earthquake Ground Motions. The earthquake ground motions are characterized in terms of peak ground acceleration, velocity, or displacement values, and seismic response spectra or acceleration time histories. For the evaluation of arch dams, the response spectrum and/or time-history representation of earthquake ground motions should be used. The ground motion parameters for the OBE are determined based on the probabilistic method. For the MCE, however, they are normally estimated by deterministic analysis, but a probabilistic analysis should also be considered so that the likelihood of a given intensity of ground motion during the design life of the dam structure can be determined. The earthquake ground motions required as input for the seismic analysis of arch dams are described in the following subparagraphs.

a. Design Response Spectra. The ground motion used for the seismic analysis of arch dams generally is defined in the form of smooth response spectra and the associated acceleration time histories. In most cases site-specific response spectra are required, except when the seismic hazard is very low; in which case a generic spectral shape such as that provided in most building codes may suffice. When site-specific response spectra are required, the effects of magnitude, distance, and local geological conditions on the amplitude and frequency content of the ground motions should be considered. In general, the shape of the response spectrum for an OBE event is different from that for the MCE, due to differences in the magnitude and the earthquake sources as shown in Figure 7-1. Thus, two separate sets of smooth response spectra may be required, one for the OBE and another for the MCE. The smooth response spectra for each design earthquake should be developed for both horizontal and vertical components of the ground motion. The design spectra are typically developed for 5 percent damping. Estimates for other damping values can be obtained using available relationships (Newmark and Hall 1982). The vertical response spectra can be estimated using the simplified published relationships between the vertical and horizontal spectra which will be described in a future engineer manual. The relationship used should recognize the significant influence of the source-to-site distance and of the particular period range ($\leq$ 0.2 sec) on the vertical response spectra.

b. Acceleration Time Histories. When acceleration time histories of ground motions are used as seismic input for the dynamic analysis of arch dams, they should be established with the design response spectra and should have appropriate strong motion duration and number of peaks. The duration of strong motion is commonly measured by the *bracketed duration.* This is the duration of shaking between the first and last accelerations of the accelerogram exceeding 0.05 g.

(1) Acceleration time histories are either selected from recorded ground motions appropriate to the site, or they are synthetically developed or modified from one or more ground motions. In the first approach, several records are usually required to ensure that the response spectra of all records as a whole do not fall below the smooth design spectra. This procedure has the advantage that the dam is analyzed for natural motions, several dynamic analyses should be performed. In addition, the response spectrum of individual records may have peaks that substantially exceed the design response spectra.

(2) Alternatively, acceleration time histories are developed either by artificially generating an accelerogram or by modifying a recorded accelerogram so that the response spectrum of the resulting accelerogram closely matches the design response spectra. The latter technique is preferred,

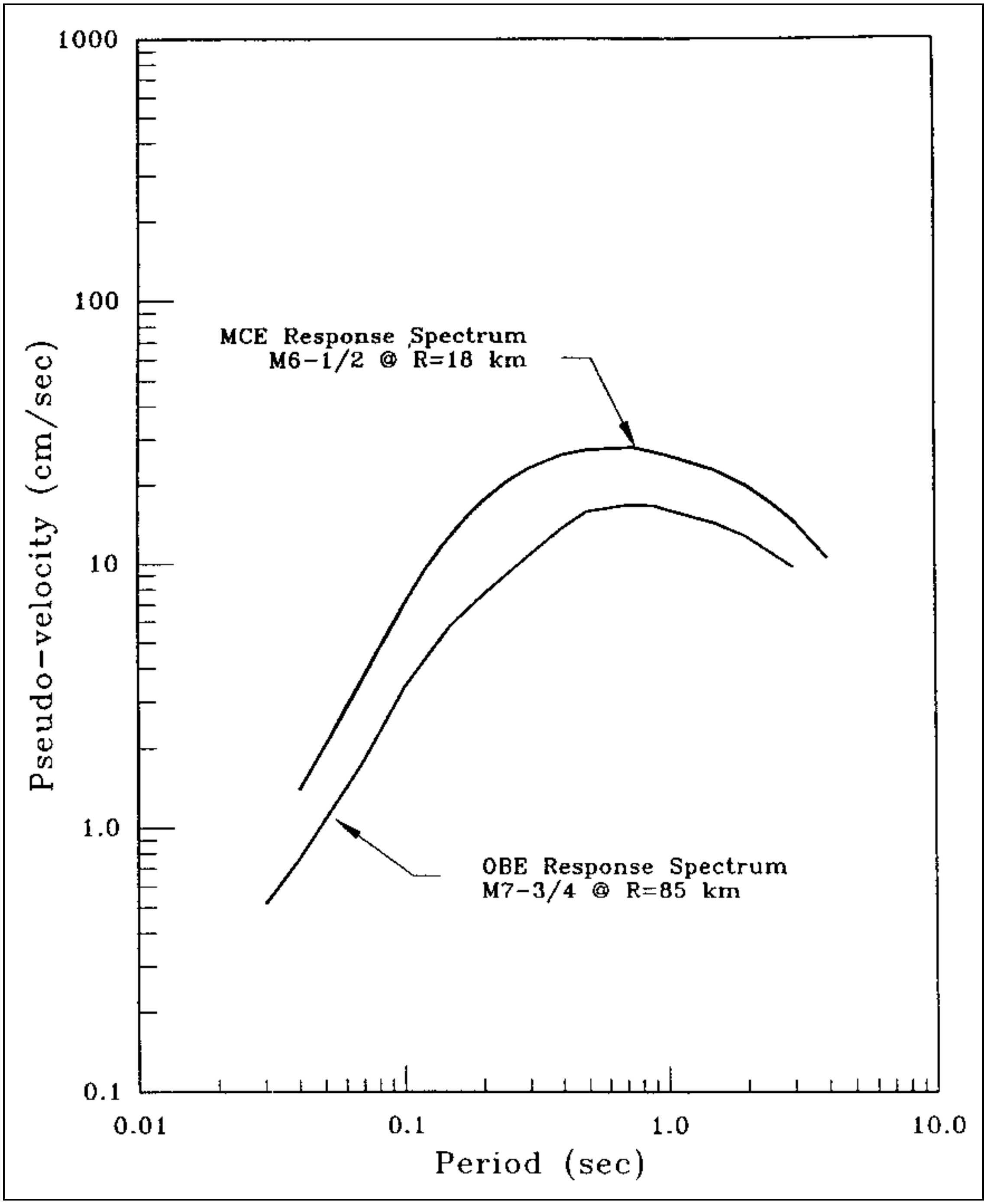


Figure 7-1. Smooth design response-spectrum examples for OBE and MCE events for 5 percent damping

because it starts with a natural accelerogram and thus preserves the duration and phasing of the original record and produces time histories that look natural. An example of this procedure which shows a good match with the smooth design response spectrum is demonstrated in Figure 7-2.

(3) For large thin arch dams with fundamental periods near 0.5 to 1 sec located at close distances to the earthquake source, it is desirable to include a strong intermediate-to-long period pulse (0.5 to 5 sec) to account for the "fling" characteristic of near-source ground motion.

7-5. Finite Element Modeling Factors Affecting Dynamic Response. Dynamic analysis of arch dams for earthquake loading should be based on a 3-D idealization of the dam-water-foundation system which accounts for the significant interaction effects of the foundation rock and the impounded water. To compute the linear response of the dam, the concrete arch and the foundation rock are modeled by standard finite elements, whereas the interaction effects of the impounded water can be represented with any of three different level of refinement. In addition, the dynamic response of arch dams is affected by the damping and by the intensity and spatial variation of the seismic input. These factors and the finite element modeling of various components of an arch dam are discussed in the following sections.

a. Arch Dam. The finite element model of an arch dam for dynamic analysis is essentially identical to that developed for the static analysis. In a linear-elastic analysis, the arch dam is modeled as a monolithic structure with no allowance for the probable contraction joint opening during earthquake excitation. Thin and moderately thin arch dams are adequately modeled by a single layer of shell elements, whereas thick gravity-arch dams should be represented by two or more layers of solid elements through the dam thickness. The size of the mesh should be selected following the general guidelines presented in Chapter 6 for static analysis and shown in Figure 6-1. In addition, the dynamic response of the appurtenant structures attached to the dam may be significant and also should be considered. For example, the power intakes attached to the dam may include free-standing cantilevers that could vibrate during the earthquake shaking. The power intakes in this case should be included as part of the dam model to ensure that the dam stresses induced by the vibration of these components are not excessive.

b. Dam-foundation Rock Interaction. Arch dams are designed to resist the major part of the water pressures and other loads by transmitting them through arch action to the canyon walls. Consequently, the effects of foundation rock on the earthquake response of arch dams are expected to be significant and must be considered in the dynamic analysis. However, a complete solution of the dam foundation interaction effects is very complicated and such procedures have not yet been fully developed. There are two major factors contributing to this complex interaction problem. First is the lack of a 3-D model of the unbounded foundation rock region to account for energy loss due to the radiation of vibration waves. The other and even more important contributing factor is related to the prescription of spatial variation of the seismic input at the dam-foundation interface, resulting from wave propagation of seismic waves through the foundation rock and from scattering by the canyon topography. Faced with these difficulties, an overly simplified model of the foundation rock (Clough 1980) is currently used in practice. This widely used simplified model ignores inertial and damping effects and considers only the

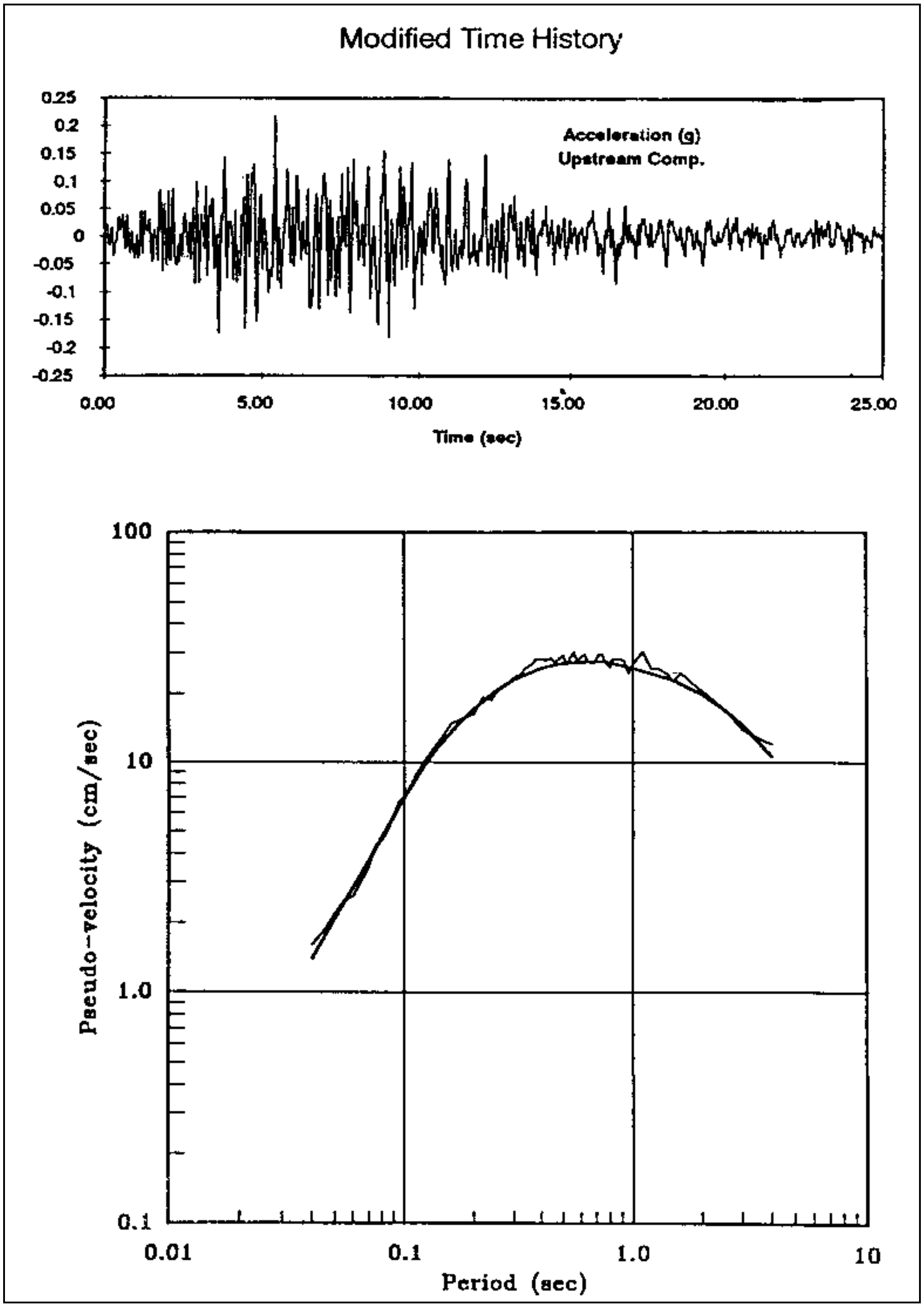


Figure 7-2. Comparison of response spectrum of modified time history and smooth-design response spectrum for 5 percent damping

flexibility of the foundation rock. The foundation model for the dynamic analysis is therefore similar to that described for the static analysis in Chapter 6. As shown in Figures 6-1a and 6-3, an appropriate volume of the foundation rock should be idealized by the finite element discretization of the rock region. Each foundation element is represented by a solid element having eight or more nodes and characterized by its dynamic deformation modulus and Poisson’s ratio.

(1) Shape of Foundation Model. Using the finite element procedure, a foundation model can be developed to match the natural topography of the foundation rock region. However, such a refined model is usually not required in practice. Instead, a prismatic model employed in the GDAP program and described in Chapter 6 may be used. This foundation model depicted in Figure 6-1a is constructed on semicircular planes cut into the canyon walls normal to the dam-foundation contact surface; in moving from the base to the dam crest, each semicircle is rotated about a diameter always oriented in the upstream-downstream direction.

(2) Size of Foundation Model. The size of the foundation model considered in GDAP is controlled by the radius (R_f) of the semicircular planes described in the previous paragraph. In the static analysis discussed previously, R_f was selected so that the static displacements and stresses induced in the dam were not changed by further increase of the foundation size. In the dynamic analysis, the natural frequencies and mode shapes of vibration control the dam response to earthquakes. Therefore, the size of a foundation model should be selected so that the static displacements and stresses, as well as the natural frequencies and mode shapes, are accurately computed. The natural frequencies of the dam-foundation system decrease as the size of the flexible foundation rock increases (Clough et al. 1985 and Fok and Chopra 1985), but for the massless foundation, the changes are negligible when the foundation size R_f is greater than one dam height, except for the foundation rocks with very low modulus of elasticity. For most practical purposes, a massless foundation model with R_f equal to one dam height is adequate. However, when the modulus ratio of the rock to concrete is less than one-half, a model with R_f equal to two times dam height should be used.

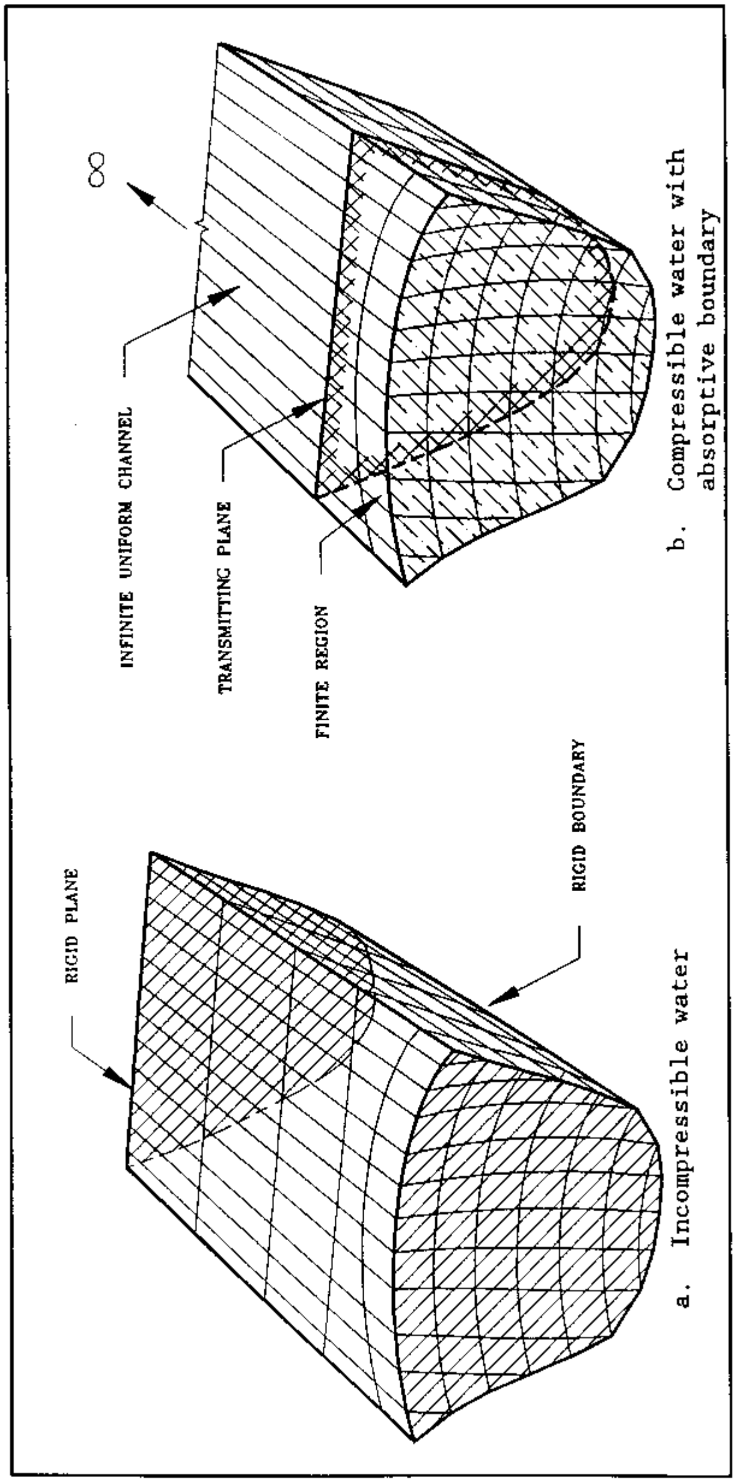
c. Dam-water Interaction. Interaction between the dam and impounded water is an important factor affecting the dynamic response of arch dams during earthquake ground shaking. In the simplest form, this interaction can be represented by an "added mass" attached to the dam first formulated by Westergaard (1933). A more accurate representation of the added mass is obtained using a finite element formulation which accounts for the complicated geometry of the arch dam and the reservoir (Kuo 1982 (Aug)). Both approaches, however, ignore compressibility of water and the energy loss due to radiation of pressure waves in the upstream direction and due to reflection and refraction at the reservoir bottom. These factors have been included in a recent and more refined formulation (Fok and Chopra 1985 (July)), but computation of the resulting frequency-dependent hydrodynamic pressure terms requires extensive efforts and requires consideration of a range of reservoir-bottom reflection coefficients.

(1) Generalized Westergaard Added Mass. Westergaard (1933) demonstrated that the effects of hydrodynamic pressures acting on the vertical face of a rigid gravity dam could be represented by an added mass attached to the

dam, if the compressibility of water is neglected. A general form of this incompressible added-mass concept has been applied to the analysis of arch dams (Kuo 1982 (Aug)). This generalized formulation, also described by Ghanaat (1993b), is based on the same parabolic pressure distribution in the vertical direction used by Westergaard, but it recognizes the fact that the hydrodynamic pressures acting on the curved surface of an arch dam are due to the total accelerations normal to the dam face. Although the resulting added mass calculated in this manner is often used in the analysis of arch dams, it does not properly consider the hydrodynamic effects. In fact, there is no rational basis for the assumed parabolic pressure distribution used for the arch dams, because limitations imposed in the original Westergaard formulation are violated. The original Westergaard formulation assumed a rigid dam with a vertical upstream face and an infinite reservoir. However, the procedure is very simple and provides a reasonable estimate of the hydrodynamic effects for preliminary or feasibility analysis. The generalized added-mass formulation has been implemented in the GDAP program and is available as an option. The program automatically calculates the added mass for each nodal point on the upstream face of the dam; the resulting added mass of water is then added to the mass of concrete to account for the hydrodynamic forces acting on the dam.

(2) Incompressible Finite Element Added Mass. In more refined analyses of new and existing arch dams, the effects of reservoir-water interaction due to seismic loading is represented by an equivalent added mass of water obtained from the hydrodynamic pressures acting on the face of the dam. The procedure is based on a finite element solution of the pressure wave equation subjected to appropriate boundary conditions (Kuo 1982 (Aug) and Ghanaat (1993b)). The nodal point pressures of the incompressible water elements are the unknowns. The bottom and sides of the reservoir, as well as a vertical plane at the upstream end, are assumed to be rigid. In addition, the hydrodynamic pressures at the water-free surface are set to zero; thus the effects of surface waves are neglected, but these have little effect on the seismic response. In general, a finite element model of the reservoir water can be developed to match the natural canyon topography, but a prismatic reservoir model available in the GDAP program is quite adequate in most practical situations. The GDAP reservoir model is represented by a cylindrical surface generated by translating the dam-water interface nodes in the upstream direction as shown in Figure 7-3a. The resulting water nodes generated in this manner match those on the dam face and are usually arranged in successive planes parallel to the dam axis, with the distance between the planes increasing with distance from the dam. Experience shows that the reservoir water should include at least three layers of elements that extend upstream a distance at least three times the water depth.

(3) Compressible Water with Absorptive Reservoir Bottom. The added-mass representation of hydrodynamic effects ignores both water compressibility effects and the energy absorption mechanism at the reservoir bottom. These factors have been included in a recent formulation of the dam-water interaction mechanism which is fully described by Fok and Chopra (1985 (July)). It introduces frequency-dependent hydrodynamic terms in the equations of motion that can be interpreted as an added mass, an added damping, and an added force. The added damping term arises from the refraction of hydrodynamic pressure waves into the absorptive reservoir bottom and also from the propagation of pressure waves in the upstream direction. The energy loss at the reservoir bottom is approximated by the wave reflection coefficient α, which



a. Incompressible water

b. Compressible water with absorptive boundary

Figure 7-3. Finite element models of fluid domain with and without water compressibility is defined as the ratio of reflected-to-incident wave amplitude of a pressure wave striking the reservoir bottom. The values of α can be varied from α = 1.0, for a rigid, nonabsorptive boundary similar to that used in the GDAP model, to α = 0.0, indicating total absorption.

(a) The response analysis of an arch dam including the effects of dam-water interaction, water compressibility, and reservoir-bottom absorption can be performed using the EACD-3D program (Fok, Hall, and Chopra 1986 (July)). The finite element idealizations of the dam and foundation rock employed in this program are essentially equivalent to those employed by the GDAP program; the fluid region near the dam is modeled by liquid finite elements similar to those in GDAP, but, unlike the GDAP, these elements are of compressible water and are connected to a uniform channel extending to infinity to permit pressure waves to radiate away from the dam (Figure 7-3b).

(b) Another major difference of the EACD-3D model is that the reservoir boundary is absorptive and thus dissipation of hydrodynamic pressure waves in the reservoir bottom materials is permitted. However, this method requires considerable computational effort and is too complicated for most practical applications. An even more important consideration is the lack of guidance or measured data for determining an appropriate α factor for use in the analysis. Consequently, such analyses must be repeated for a range of α factors in order to establish a lower and upper bound estimate of the dam response. It is also important to note that the significance of water compressibility depends on the dynamic characteristics of the dam and the impounded water. Similar to gravity dams (Chopra 1968), the effects of water compressibility for an arch dam can be neglected if the ratio of the natural frequency of the reservoir water to the natural frequency of the arch dam-foundation system without water is greater than 2.

d. Damping. Damping has a significant effect on the response of an arch dam to earthquake and other dynamic loads. The energy loss arises from several sources including the concrete arch structure, foundation rock, and the reservoir water. Dissipation of energy in the concrete arch structure is due to internal friction within the concrete material and at construction joints. In the foundation rock this energy loss is facilitated by propagation of elastic waves away from the dam (radiation damping) and by hysteretic losses due to sliding on cracks and fissures within the rock volume. An additional source of damping, as discussed in paragraph 7-5c(3), is associated with the energy loss due to refraction of hydrodynamic pressure waves into the reservoir bottom materials and propagation of pressure waves in the upstream direction.

(1) The current standard earthquake analysis of arch dams is based on a massless foundation rock model and employs incompressible added mass for representing the hydrodynamic effects. In this type of analysis, only the material damping associated with the concrete structure is explicitly considered. The overall damping constant for the entire model in such linear-elastic analyses is normally specified based on the amplitude of the displacements, the opening of the vertical contraction joints, and the amount of cracking that may occur in the concrete arch. Considering that the measured damping values for concrete dams subjected to earthquake loading are scarce and that the effects of contraction joints, lift surfaces, and cracks cannot

be precisely determined, the damping value for a moderate shaking such as an OBE event should be limited to 5 percent.

(2) However, under the MCE earthquake ground motions, damping constants of 7 or 10 percent may be used depending on the level of strains developed in the concrete and the amount of nonlinear joint opening and/or cracking that occurs. In more severe MCE conditions, especially for large dams, additional damping can be incorporated in the analysis by employing a dam-water interaction model which includes water compressibility and permits for the dissipation of energy at the reservoir boundary.

7-6. Method of Analysis. The current earthquake response analysis of arch dams is based on linear-elastic dynamic analysis using the finite element procedures. It is assumed that the concrete dam and the interaction mechanisms with the foundation rock and the impounded water exhibit linear-elastic behavior. Using this method, the arch dam and the foundation rock are treated as 3-D systems idealized by the finite element discretization discussed in previous paragraphs and in Chapter 6. Under the incompressible added-mass assumption for the impounded water, the response analysis is performed using the response-spectrum modal-superposition or the time-history method. For the case of compressible water, however, the response of the dam to dynamic loads must be evaluated using a frequency-domain procedure, in order to deal with the frequency-dependent hydrodynamic terms. These methods of analyses are discussed in the following paragraphs.

a. Response-spectrum Analysis. The response-spectrum method of analysis uses a response-spectrum representation of the seismic input motions to compute the maximum response of an arch dam to earthquake loads. This approximate method provides an efficient procedure for the preliminary analyses of new and existing arch dams. It may also be used for the final analyses, if the calculated maximum stress values are sufficiently less than the allowable stresses of the concrete. Using this procedure, the maximum response of the arch dam is obtained by combining the maximum responses for each mode of vibration computed separately.

(1) A complete description of the method is given in the theoretical manual by Ghanaat (1993b). First, the natural frequencies and mode shapes of undamped free vibration for the combined dam-water-foundation system are evaluated; the free vibration equations of motion are assembled considering the mass of the dam-water system and the stiffness of the combined dam and foundation rock models. The maximum response in each mode of vibration is then obtained from the specified response spectrum for each component of the ground motion, using the modal damping and the natural period of vibration for each particular mode. The same damping constant is used in all modes as represented by the response- spectrum curves. Since each mode reaches its maximum response at a different time, the total maximum response quantities for the dam, such as the nodal displacements and the element stresses, are approximated by combining the modal responses using the square root of the sum of the squares (SRSS) or complete quatratic combination (CQC) procedure. Finally, the resulting total maximum responses evaluated independently for each component of the earthquake ground motion are further combined by the SRSS method for the three earthquake input components, two horizontal and one vertical.

(2) For a linear-elastic response, only a few lower modes of vibration are needed to express the essential dynamic behavior of the dam structure. The appropriate number of vibration modes required in a particular analysis depends on the dynamic characteristics of the dam structure and on the nature of earthquake ground motion. But, in all cases, a sufficient number of modes should be included so that at least 90 percent of the "exact" dynamic response is achieved. Since the "exact" response values are not known, a trial-and-error procedure may be adapted, or it may be demonstrated that the participating effective modal masses are at least 90 percent of the total mass of the structure.

b. Time-history Analysis. Time-history analysis should be performed when the maximum stress values computed by the response-spectrum method are approaching or exceeding the tensile strength of the concrete. In these situations, linear-elastic time-history analyses are performed to estimate the maximum stresses more accurately as well as to account for the time-dependent nature of the dynamic response. Time-history analyses provide not only the maximum stress values, but also the simultaneous, spatial extent and number of excursions beyond any specified stress value. Thus, they can indicate if the calculated stresses beyond the allowable values are isolated incidents or if they occur repeatedly and over a significant area.

(1) The seismic input in time-history analyses is represented by the acceleration time histories of the earthquake ground motion. Three acceleration records corresponding to three components of the specified earthquake are required; they should be applied at the fixed boundaries of the foundation model in the channel, across the channel, and in the vertical directions. The acceleration time histories are established following the procedures described in paragraph 7-4b.

(2) The structural models of the dam, foundation rock, and the impounded water for a time-history analysis are identical to those developed for response-spectrum analysis. However, the solution to the equations of motion is obtained by a step-by-step numerical integration procedure. Two methods of solution are available: direct integration and mode superposition (Ghanaat, technical report in preparation). In the direct method, step-by-step integration is applied to the original equations of motion with no transformation being carried out to uncouple them. Hence, this method requires that the damping matrix to be represented is in explicit form. In practice, this is accomplished using Raleigh damping (Clough and Penzien 1975), which is of the form

$$c = a_0\, m + a_1\, k$$

where coefficients a_0 and a_1 are obtained from two given damping ratios associated with two frequencies of vibration. The direct integration method is most effective when the response is required for a relatively short duration. Otherwise, the mode superposition method in which the step-by-step integration is applied to the uncoupled equations of motion will be more efficient. In the mode superposition method, first the undamped vibration mode shapes and frequencies are calculated, and the equations of motion are transformed to those coordinates. Then the response history for each mode is evaluated separately at each time-step, and the calculated modal response histories are combined to obtain the total response of the dam structure. It should be noted that the damping in this case is expressed by the modal damping ratios and need not be specified in explicit form.

7-7. Evaluation and Presentation of Results. The earthquake performance of arch dams is currently evaluated using the numerical results obtained from a linear-dynamic analysis. The results of linear analysis provide a satisfactory estimate of the dynamic response to low- or moderate-intensity OBE earthquake motions for which the resulting deformations of the dam are within the linear-elastic range. In this case, the performance evaluation is based on simple stress checks in which the calculated elastic stresses are compared with the specified strength of the concrete. Under the MCE ground motions, it is possible that the calculated stresses would exceed the allowable values and that significant damage could occur. In such extreme cases, the dam should retain the impounded water without rupture, but the actual level of damage can be estimated only by a nonlinear analysis that takes account of the basic nonlinear behavior mechanisms such as the joint opening, tensile cracking, and the foundation failure. However, a complete nonlinear analysis is not currently possible, and linear analysis continues to be the primary tool for assessing the seismic performance of arch dams subjected to damaging earthquakes. Evaluation of the seismic performance for the MCE is more complicated, it requires some judgement and elaborate interpretations of the results before a reasonable estimate of the expected level of damage can be made or the possibility of collapse can be assessed.

a. Evaluation of Response-spectrum Analysis. The first step in response spectrum analysis is the calculation of vibration mode shapes and frequencies. The mode shapes and frequencies provide insight into the basic dynamic response behavior of an arch dam. They provide some advance indication of the sensitivity of the dynamic response to earthquake ground motions having various frequency contents. Figure 7-4 demonstrates a convenient way for presenting the mode shapes. In this figure the vibration modes are depicted as the plot of deflected shapes along the arch sections at various elevations. After the calculation of mode shapes and frequencies, the maximum dynamic response of the dam structure is computed. These usually include the maximum nodal displacements and element stresses. In particular, the element stresses are the primary response quantity used for the evaluation of earthquake performance of the dam.

(1) Dynamic Response. The basic results of a response-spectrum analysis include the extreme values of the nodal displacements and element stresses due to the earthquake loading. As discussed earlier, these extreme response values are obtained by combining the maximum responses developed in each mode of vibration using the SRSS or CQC combination rule. In addition, they are further combined by the SRSS method to include the effects of all three components of the earthquake ground motion. Thus, the resulting dynamic response values obtained in this manner have no sign and should be interpreted as being either positive or negative. For example the response-spectrum stress values are assumed to be either tension or compression.

(2) Total Response. The evaluation of earthquake performance of an arch dam using the response-spectrum method of analysis involves comparison of the total stresses due to both static and earthquake loads with the expected

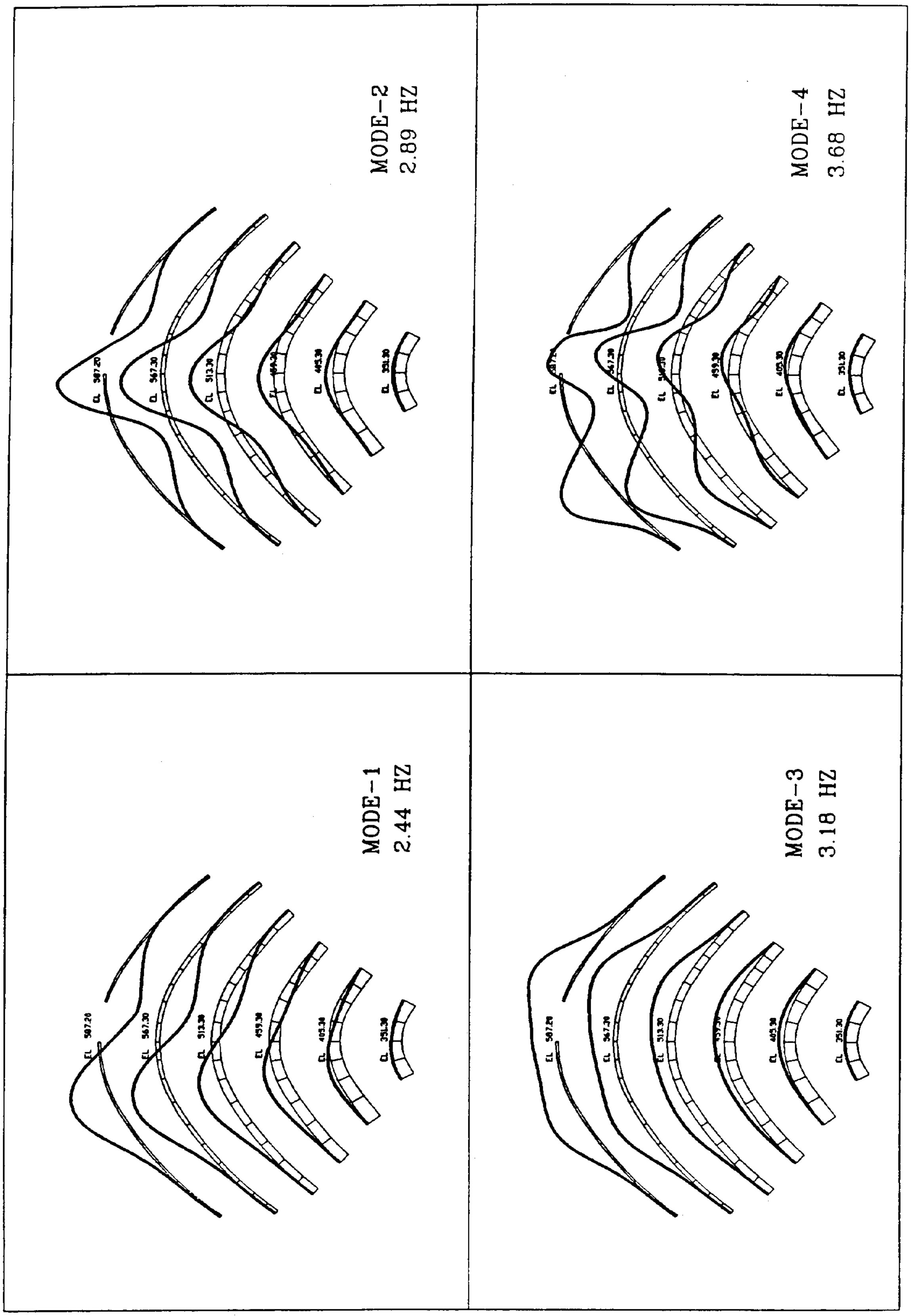


Figure 7-4. Four lowest vibration modes of Portugues Arch Dam

strength of the concrete. To obtain the total stress values, the response-spectrum estimate of the dynamic stresses (σ_d) should be combined with the effects of static loads (σ_{st}). The static stresses in a dam prior to the earthquake are computed using the procedures described in Chapter 6. The static loads to be considered include the self-weight, hydrostatic pressures, and the temperature changes that are expected during the normal operating condition as discussed in Chapter 4. Since response-spectrum stresses have no sign, combination of static and dynamic stresses should consider dynamic stresses to be either positive or negative, leading to the maximum values of total tensile or compressive stresses:

$$\sigma_{max} = \sigma_{st} \pm \sigma_d$$

(a) This combination of static and dynamic stresses is appropriate if Σ_{st} and Σ_d are oriented similarly. This is true for arch or cantilever stresses at any point on the dam surface, but generally is not true for the principal stresses. In fact, it is not possible to calculate the principal stresses from a response-spectrum analysis, because the maximum arch and cantilever stresses do not occur at the same time; therefore, they cannot be used in the principal stress formulas.

(b) The computed total arch and cantilever stresses for the upstream and downstream faces of the dam should be displayed in the form of stress contours as shown in Figure 7-5. These represent the envelopes of maximum total arch and cantilever stresses on the faces of the dam, but because they are not concurrent they cannot be combined to obtain envelopes of principal stresses, as was mentioned previously.

b. Results of Time-history Analysis. Time-history analysis computes time-dependent dynamic response of the dam model for the entire duration of the earthquake excitation. The results of such analyses provide not only the maximum response values, but also include time-dependent information that must be examined and interpreted systematically. Although evaluation of the dynamic response alone may sometimes be required, the final evaluation should be based on the total response which also includes the effects of static loads.

(1) Mode Shapes and Nodal Displacements. Vibration mode shapes and frequencies are required when the mode-superposition method of time-history analysis is employed. But it is also a good practice to compute them for the direct method. The computed vibration modes may be presented as shown in Figure 7-4 and discussed previously. The magnitude of nodal displacements and deflected shape of an arch dam provide a visual means for the evaluation of earthquake performance. As a minimum, displacement time histories for several critical nodal points should be displayed and evaluated. Figure 7-6 shows an example of such displacement histories for a nodal point on the dam crest.

(2) Envelopes of Maximum and Minimum Arch and Cantilever Stresses. Examination of the stress results for a time-history analysis should start with presentation of the maximum and minimum arch and cantilever stresses. These stresses should be displayed in the form of contour plots for the upstream and downstream faces of the dam. The contour plots of the maximum

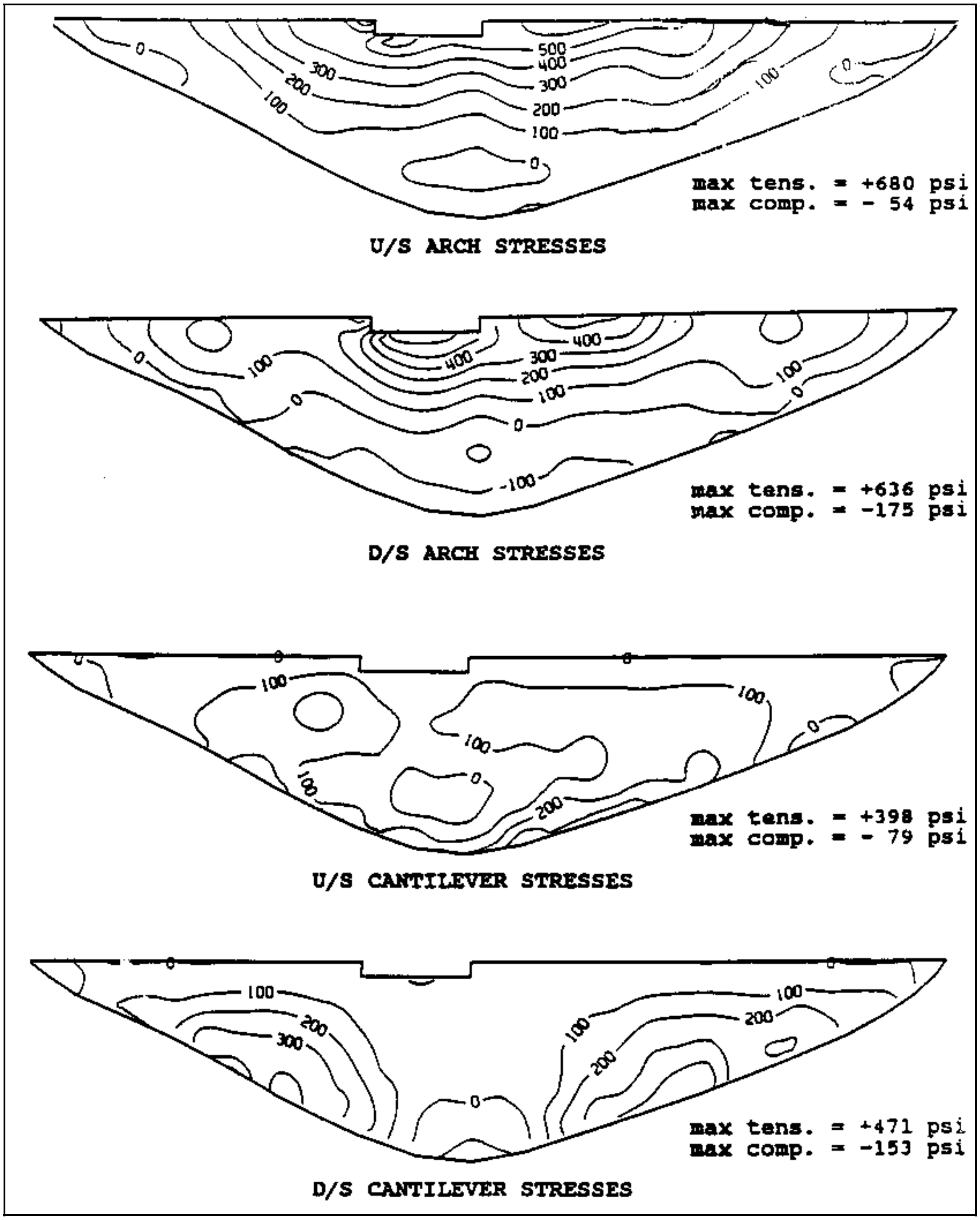


Figure 7-5. Envelope of maximum arch and cantilever stress (in psi)

arch and cantilever stresses represent the largest computed tensile (positive) stresses at all locations in the dam during the earthquake ground shaking (Figure 7-5). Similarly, the contour plots of the minimum stresses represent the largest compressive (negative) arch and cantilever stresses in the dam. The maximum and the minimum stresses at different points are generally reached at different instants of time. Contour plots of the maximum arch and cantilever stresses provide a convenient means for identifying the overstressed

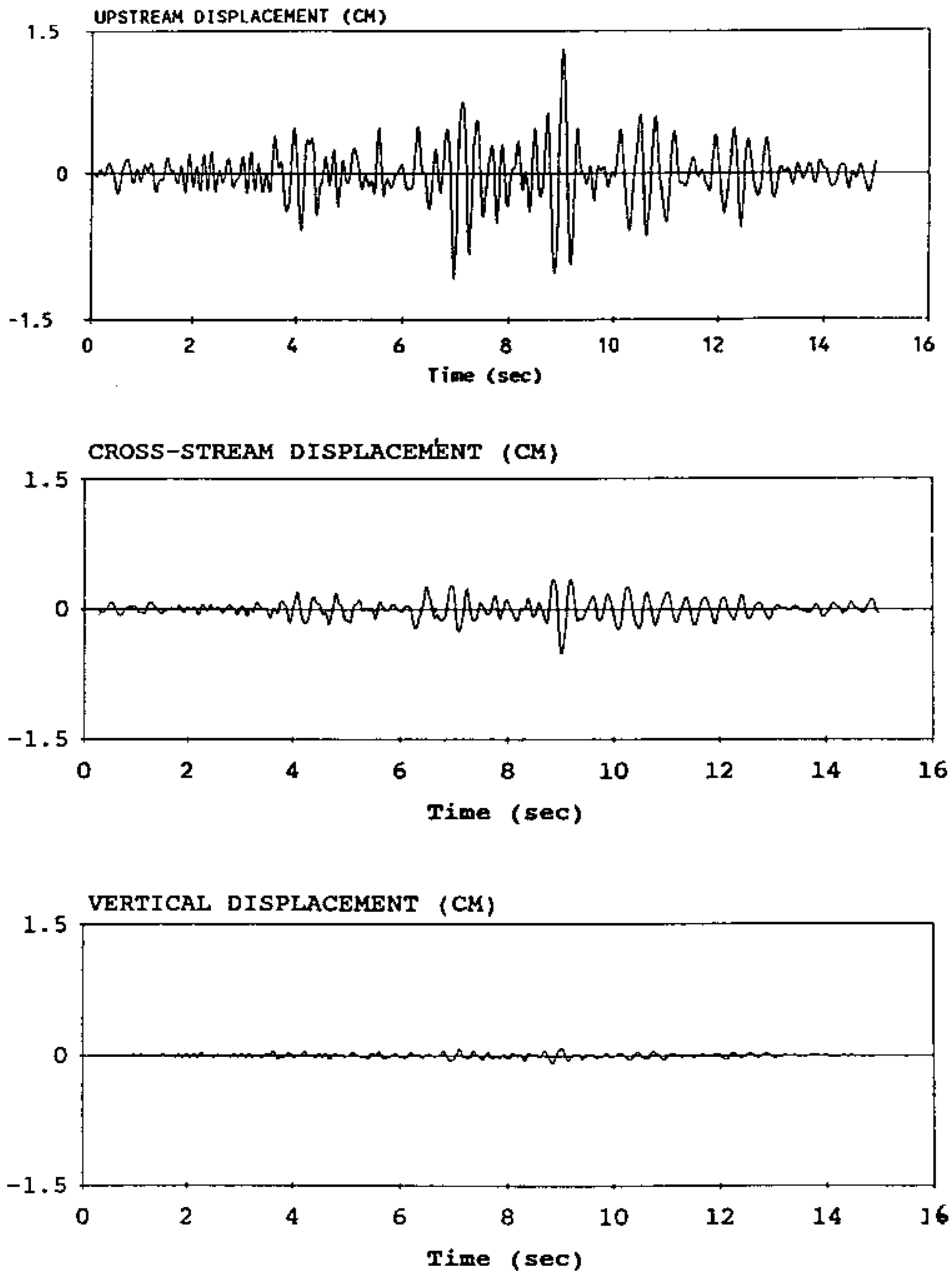


Figure 7-6. Displacement time history of a crest node in upstream, cross-stream, and vertical direction

areas where the maximum stresses approach or exceed tensile strength of the concrete. Based on this information, the extent and severity of tensile stresses are determined, and if necessary, further evaluation which accounts for the time-dependent nature of the dynamic response should be made as described in the following sections. Contour plots of the minimum stresses show the extreme compressive stresses that the dam would experience during the earthquake loading. The compressive stresses should be examined to ensure that they meet the specified safety factors for the dynamic loading (Chapter 11).

(3) Concurrent Stresses. The envelopes of maximum and minimum stresses discussed in paragraph 7-7b(2) demonstrate the largest tensile and compressive stresses that are developed at different instants of time. They serve to identify the overstressed regions and the times at which the critical stresses occur. This information is then used to produce the concurrent (or simultaneous) state of stresses corresponding to the time steps at which the critical stresses in the overstressed regions reach their maxima. The concurrent arch and cantilever stresses in the form of contour plots (Figure 7-7) can be viewed as snap shots of the worst stress conditions.

(4) Envelopes of Maximum and Minimum Principal Stresses. The time histories of principal stresses at any point on the faces of the dam are easily computed from the histories of arch, cantilever, and shear stresses at that point. When the effects of static loads are considered, the static and dynamic arch, cantilever, and shear stresses must be combined for each instant of time prior to the calculation of the total principal stresses for the same times. The resulting time histories of principal stresses are used to obtain the maxima and minima at all points on both faces of the dam which are then presented as vector plots as shown in Figures 7-8 and 7-9.

(5) Time History of Critical Stresses. When the maximum and concurrent stresses show that the computed stresses exceed the allowable value, the time histories of critical stresses should be presented for a more detailed evaluation (Figure 7-10). In this evaluation the time histories for the largest maximum arch and cantilever stresses should be examined to determine the number of cycles that the maximum stresses exceed the allowable value. This would indicate whether the excursion beyond the allowable value is an isolated case or is repeated many times during the ground motion. The total duration that the allowable value (or cracking stress) is exceeded by these excursions should also be estimated to demonstrate whether the maximum stress cycles are merely spikes or they are of longer duration and, thus, more damaging. The number of times that the allowable stress can safely be exceeded has not yet been established. In practice, however, up to five stress cycles have been permitted based on judgement but have not been substantiated by experimental data. The stress histories at each critical location should be examined for two opposite points on the upstream and downstream faces of the dam as in Figure 7-10. For example, a pair of cantilever stress histories can demonstrate if stresses on both faces are tension, or if one is tension and the other is compression. The implication of cantilever stresses being tension on both faces is that the tensile cracking may penetrate through the dam section, whereas in the case of arch stresses, this indicates a complete separation of the contraction joint at the location of maximum tensile stresses.

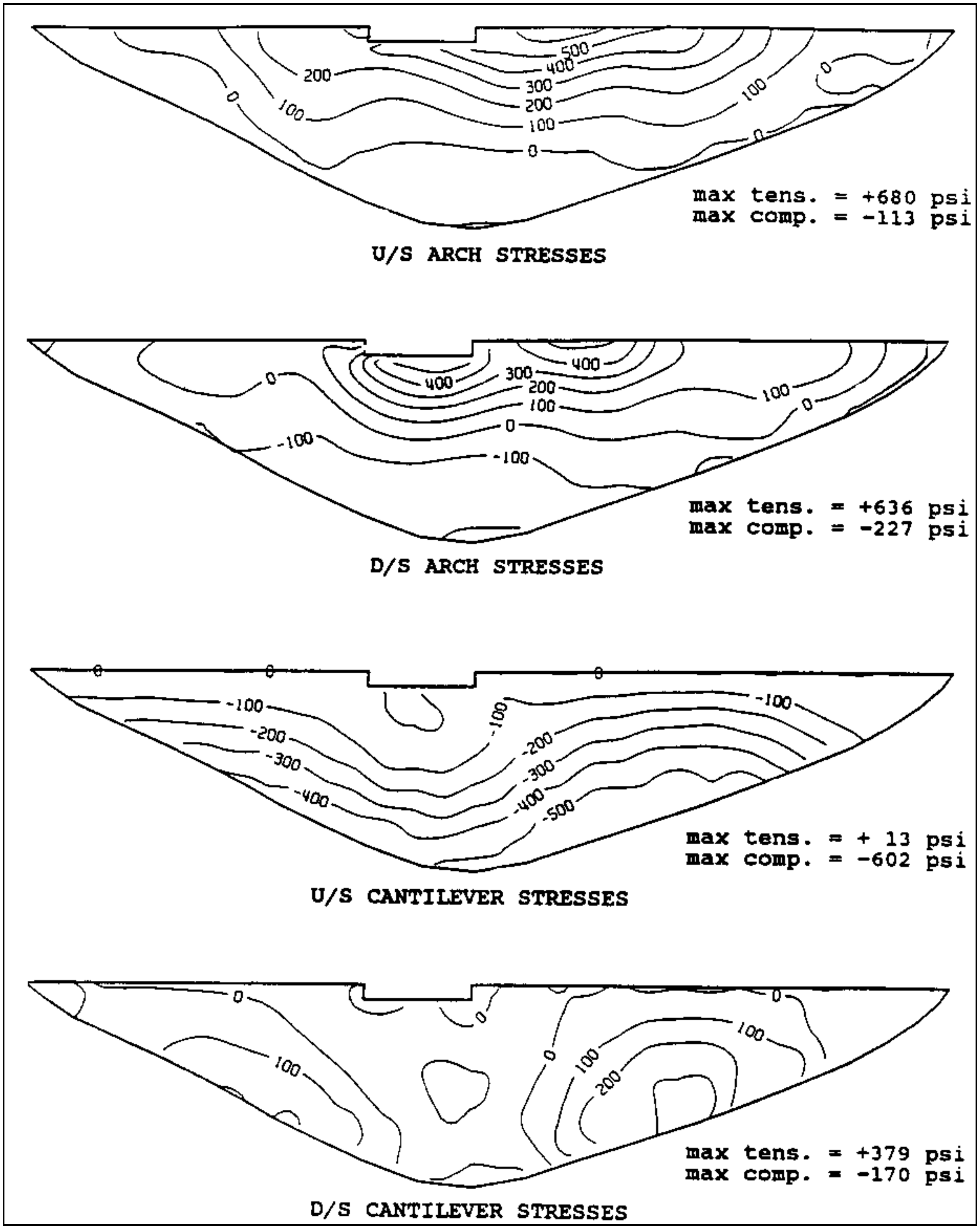


Figure 7-7. Concurrent arch and cantilever stresses (in psi) at time-step corresponding to maximum arch stress

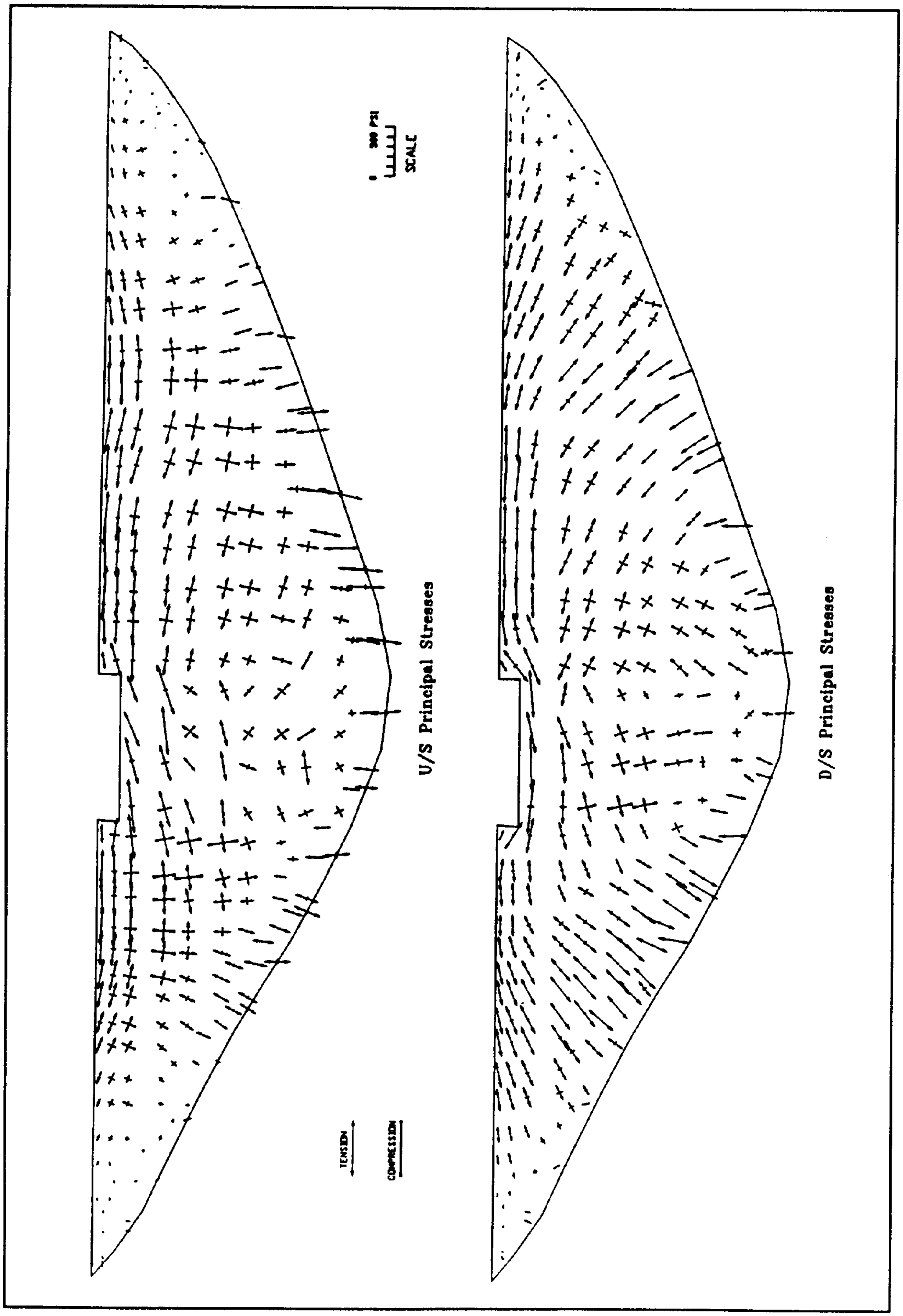


Figure 7-8. Envelope of maximum principal stresses with their corresponding perpendicular pair

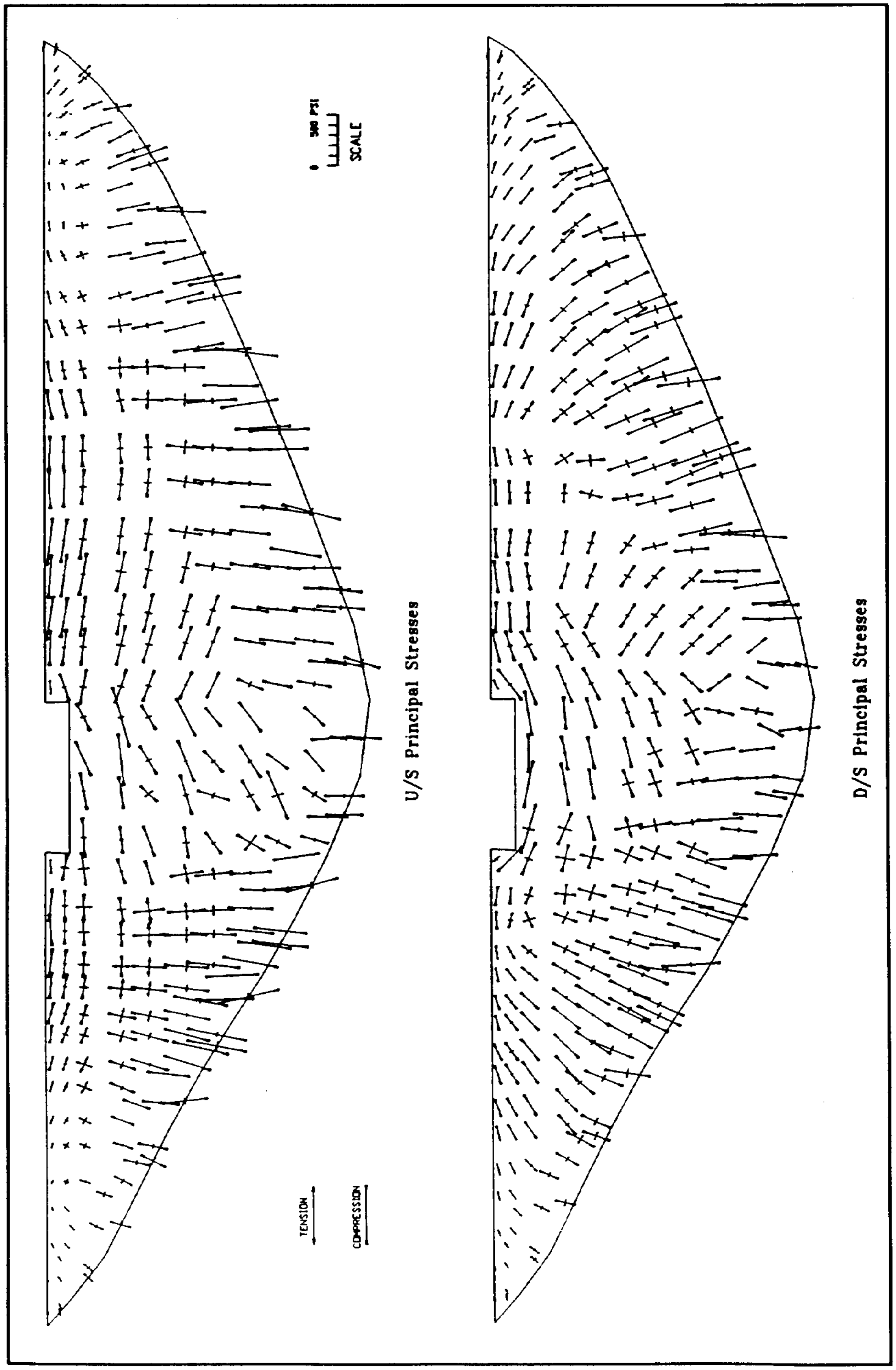


Figure 7-9. Envelope of maximum-minimum principal stresses with their corresponding perpendicular pair

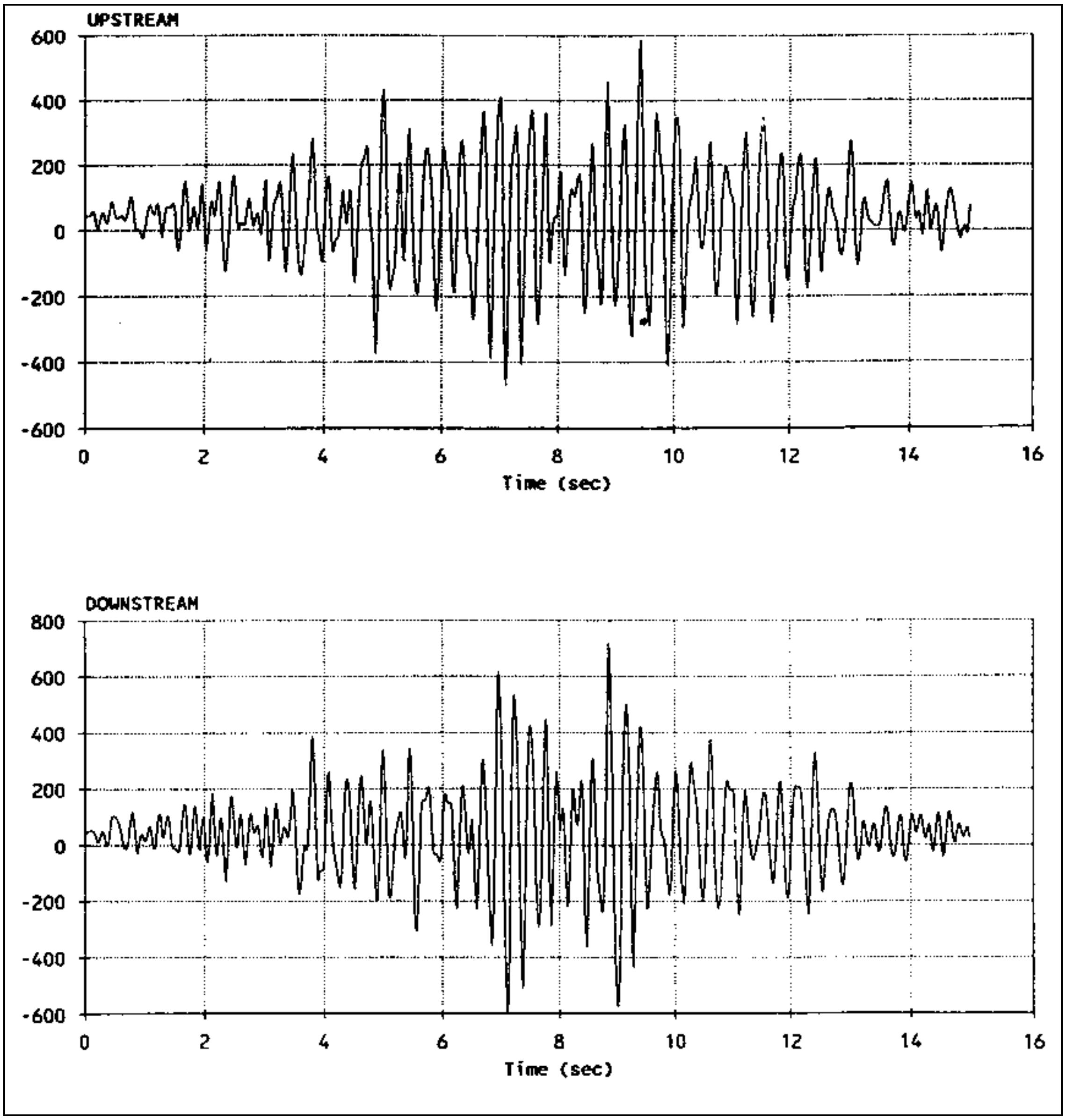


Figure 7-10. Time histories of arch stresses (in psi) at two opposite points on upstream and downstream faces of dam